2018

Bond behaviour of steel plate reinforced concrete beams

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Abstract

This technical note presents an experimental study on the bond behaviour of chequer steel plate reinforcements in concrete members based on the beam-end method. The effects of lozenges of the chequer steel plate, the use of steel bolts, and the thickness of the concrete cover on the bond behaviour are investigated. The experimental program includes five specimens designed as beam-end pullout members. Each specimen is 225 mmwide, 300 mmhigh and 600 mmlong. Stirrups with 80 mmcentre-to-centre spacing are used as confinement for all specimens. The first specimen is reinforced with a deformed steel bar whereas the remaining specimens are reinforced with steel plates. All specimens except for the one reinforced with a smooth steel plate failed by pullout accompanied by splitting crack. The lozenges of chequer steel plate increased the ultimate pullout failure load by 80% compared to that of the specimen reinforced with a smooth steel plate. It has also been found that the pullout failure load of a steel plate reinforced concrete member can be significantly affected by the thickness of the concrete cover. Two other significant findings are that the pre-ultimate slippage of a steel plate reinforced concrete member is much less than that of a deformed steel bar reinforced one, and that the post-ultimate behaviour of the former is much more ductile than the latter. Comparisons between the present test results and the earlier test results involving reinforced concrete beams subjected to four-point bending tests suggest that the beam-end method may not be an appropriate method for comparing the bond strength of a chequer steel plate against that of a reinforcing bar.

Disciplines

Engineering | Science and Technology Studies

Publication Details

Sarhan, M. M., Hadi, M. N. S. & Teh, L. H. (2018). Bond behaviour of steel plate reinforced concrete beams. Construction and Building Materials, 189 751-756.

Bond Behaviour of Steel Plate Reinforced Concrete Beams

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to that of the specimen reinforced with a smooth steel plate. It has also been found that the pullout failure load of a steel plate reinforced concrete member can be significantly affected by the thickness of the concrete cover. Two other significant findings are that the pre-ultimate slippage of a steel plate reinforced concrete member is much less than that of a deformed steel bar reinforced one, and that the post-ultimate behaviour of the former is much more ductile than the latter. Comparisons between the present test results and the earlier test results involving reinforced concrete beams subjected to four-point bending tests suggest that the beam-end method may not be an appropriate method for comparing the bond strength of a chequer steel plate against that of a reinforcing bar.

Keywords: beam-end test; concrete bond; chequer steel plate; reinforced concrete; reinforcement slippage; reinforcement toughness; steel plate reinforcement.

1. Introduction

The bond strength between concrete and its steel reinforcement is a key factor for the ultimate load-carrying capacity of a reinforced concrete member. It also influences some serviceability design issues such as crack width, crack spacing and deflection of the member [1-3]. According to ACI-408R-03 [4], the transfer of forces from a deformed reinforcement bar to the surrounding concrete takes place by (a) chemical adhesion between the bar and the concrete, which is controlled by the surface condition of the bar and the concrete type; (b) frictional forces between the bar and the concrete, which depends on the interface's roughness, normal forces on the surface of the bar, and relative slippage between the bar and the concrete; and (c) mechanical anchorage or bearing of the ribs against the concrete.

There are five well-known methods to investigate the bond between concrete and steel reinforcement bars. The first method is the direct pullout test recommended by RILEM-7-II-128 [5] and employed by Alavi-Fard and Marzouk [6], Chan et al. [7], Campione et al. [8], Fang [9], Fang et al. [10], Bamonte and Gambarova [11], Cattaneo and Rosati [12], Tastani and Pantazopoulou [13], Belarbi et al. [14], and Desnerck et al. [15]. The direct pullout method uses a concrete cylinder with a known bonded length of the bar, and can be carried out with either the concentric or eccentric position of the bar. There are several reasons for selecting this method, including the ease of fabrication, the simplicity of the test, and the ability to isolate the different parameters that have effects on the overall bond behaviour.

The second and third methods are the anchorage beam and the splice beam tests recommended by ACI-408R-03 [4], depicted in Figures 1(a) and 1(b), respectively. The anchorage beam method uses a concrete beam with a specified bonded length of the bar and two flexural splits, tested under four-point bending [16]. The splice beam method uses a concrete beam with a known bonded length of the bar and a known splice length of the bars (the splice length exists in the constant moment zone). The splice beam specimen is relatively easy to fabricate, and provides a similar bond strength to that obtained using the beam anchorage method. The splice beam method has been used by several researchers, for example Zuo and Darwin [17], Ichinose et al. [18], Mazaheripour et al. [19], Bandelt and Billington [20], and Prince et al. [21].

The fourth method is the beam-bending test introduced by RILEM-7-II-28D [22], depicted in Figure 1(c). The specimen consists of two symmetrical blocks connected to each other by a steel hinge at the top and by the reinforcement bar near the bottom. It is subjected to four-point bending during the test. It has been employed by Belarbi et al. [14], Desnerck et al.

[23], Kotynia [24], Almeida Filho et al. [25], Chikh et al. [26], Mazaheripour et al. [19], and Tutikian et al. [27].

The fifth method is the beam-end test recommended by ASTM-A944–10 [28], which uses a concrete beam with a known bonded length of the bar, as depicted in Figure 2. In order to avoid conical surface failure of the specimen, a certain length of the bar close to the beam end is unbonded by using plastic sleeves, as shown in Figure 2. The beam-end method has been used by El-Hacha et al. [29], Sofi et al. [30], Sarker [31], Sarker [32], Hongwei and Yuxi [33] and Moen and Sharp [34].

The present study investigates the bond behaviour of beams reinforced with chequer steel plates using the beam-end method recommended by ASTM-A944–10 [28]. It also provides comparisons between the present test results and those obtained by the authors for plate and bar reinforced concrete beam specimens subjected to four-point bending tests [35].

2. Experimental program

2.1 Specimen configurations and preparation

A total of five chequer-plate reinforced concrete specimens, confined with stirrups of 10-mm plain steel bars spaced at 80 mm from each other, were tested. Each concrete specimen was 225 mm wide, 300 mm high, and 600 mm long, embedding a 100 mm by 10 mm steel chequer plate over 225 mm in the manner shown in Figure 2. The specimen designations are shown in Table 1.

The first specimen (BE-N20) had a N20 steel bar (20-mm-diameter deformed steel bar of 500 MPa nominal yield stress), as shown in Figure 2(a). Each of the remaining four specimens

(BE-HP, BE-HSP, BE-HBP, and BE-VP) had a chequer steel plate of a yield stress between 330 and 390 MPa. The steel plate was installed horizontally in Specimen BE-HP, as shown in Figure 2(b). In Specimen BE-HSP, the steel plate had two smooth faces as the lozenges were removed, as indicated in Figure 2(c). Specimen BE-HBP had a steel bolt of 20 mm diameter and 100 mm length welded to the steel plate (on the smooth face) at the mid-distance of the embedded length, as shown in Figure 2(d). The nominal yield stress of the steel bolt was 460 MPa. Specimen BE-VP had the same details as Specimen BE-HP except that the steel plate was embedded vertically, as shown in Figure 2(e).

Figure 3 shows the geometry of the lozenges in the chequer steel plates used in the present study. The plate had a regular pattern of raised lozenges on one of the two faces, the reverse face was smooth (featureless face). Each lozenge was 5.5 mm wide, 26 mm long, and 1.5 mm high. The perpendicular distance between any two parallel neighbouring lozenges was 22.5 mm, and the lozenges came in two right angle directions.

The lozenges of the chequer steel plate for Specimen BE-HSP were removed using a grinder, resulting in a featureless surface as shown in Figure 4(a). The steel bolt in Specimen BE-HBP was completely welded around its circumference to the smooth surface of the chequer steel plate, as shown in Figure 4(b).

The steel bar and chequer steel plates were unbonded by using PVC pipes and PVC tapes, respectively. Silicone glue was used at the ends (circumferences) of the unbonded areas to prevent the encroachment of concrete. Steel wires were used to fasten the stirrups to the longitudinal steel bars. Steel chairs having a height of 20 mm were placed under the stirrups to provide the bottom cover for each specimen. Steel screws were placed on the bottom of the formwork to prevent horizontal movement of the chequer steel plate during concrete casting.

The interior surfaces of the formwork and the reinforcements were cleaned from dust using compressed air prior to casting the concrete. A ready-mix concrete with a maximum aggregate size of 10 mm was used. To remove air bubbles from the concrete, an electrical vibrator was used for each specimen. The specimens were cured by keeping them wet using Hessian rugs and plastic sheets for 28 days.

2.2 Material properties

For the purpose of determining the concrete compressive strength, concrete cylinders were cast based on Australian Standards 1012.9-1999 [36], 100 mm in diameter and 200 mm in height. The concrete cylinders were cured in a water tank until the respective days of the tests. The compressive strengths, each as the average of three samples, were 32.6, 42.3, and 49.2 MPa at 7, 28, and 56 days, respectively.

In order to obtain the indirect tensile strength of concrete, concrete cylinders were cast according to Australian Standards 1012.10-2000 [37], 150 mm in diameter and 300 mm in height. The indirect tensile strength of concrete was found to be 3.5 MPa.

Three 500-mm long samples of both the plain (R10) and the deformed (N20) steel bars were tested in tension according to Australian Standards 1391-2007 [38] using a 500-kN Instron universal testing machine. The average yield stress of the plain bar was found to be 365 MPa, and that of the deformed bar was 540 MPa. The corresponding tensile strengths were 490 MPa and 625 MPa, respectively.

Five tension coupons of the chequer steel plates, each being 80 mm wide and 500 mm long, were also tested according to Australian Standards 1391-2007 [38]. The average yield stress was found to be 370 MPa and the tensile strength was 484 MPa.

2.3 Test procedure

The beam-end specimens were tested in the manner depicted in Figure 5. The tests were carried out by using the 600 kN actuator. Each beam-end specimen was placed on a steel beam and partially capped at the top with a 25-mm thick steel plate. The concrete beam-end specimen was thus anchored to the steel beam by running two 28-mm steel threaded rods through itself between the steel beam flange and the cap steel plate, secured with nuts. Two supports were used to restrain the specimens in the horizontal direction, as indicated in Figure 5.

All the tests were carried out under a displacement controlled loading regime at the stroke rate of 1 mm/minute. The applied axial tension load and the displacement were recorded through an internal load cell. Each beam-end pullout specimen was loaded until the pullout failure, which was observed as a decrease in the applied load with an increase in the displacement.

3. Experimental results and discussions

Except for the specimen reinforced with a smooth steel plate (Specimen BE-HSP), the failure mode involved pullout of the embedded steel plate or bar and splitting crack of the concrete along the embedded length, as shown in Figure 6. The surface cracks were observed after the respective ultimate test loads were reached, starting from the anchorage end on the soffit side and propagating towards the loaded end. For each of Specimens BE-HP, BE-HBP and BE-VP, a wedge formed between the soffit and one of the two adjoining sides. On the other hand, no visible cracks were observed for Specimen BE-HSP, which failed by pullout of the plate only.

A high level of confinement was provided in these beam-end specimens by the transverse reinforcement. The confinement constrained the progress of splitting cracks, produced a significant increase in the ultimate load, and affected the failure mode. The R10 stirrup bars acted as shear reinforcements during crack propagation and therefore presented more ductile behaviour of the specimens. No yield or rupture of the steel bar or chequer steel plate was observed for any of the specimens. The behaviour of the present beam-end specimens was consistent with that found by Zuo and Darwin [17] and El-Hacha et al. [29].

Figure 7 shows the load-displacement graphs of the present beam-end specimens. The peak pullout loads of Specimens BE-N20, BE-HP, BE-HSP, BE-HBP, and BE-VP were 176, 99, 55, 127, and 199 kN, respectively. It is interesting to note that, prior to the ultimate limit state, the slippage of each of the plate reinforcements was much smaller than that of the deformed bar reinforcement. The reason is that the bond area of each steel plate was much larger than that of the steel bar.

It can also be seen from the results of Specimens BE-HP and BE-HSP that the lozenges of the chequer steel plate increased the bond load by 80%, emphasising the benefit of using chequer steel plates rather than plain steel plates for concrete reinforcement.

The result of Specimen BE-VP points to the very significant effect of the concrete cover's thickness on the bond strength. Further research is required to quantify such an effect in terms of the cover thickness.

A significant outcome of the present test results is that all the steel plate reinforcements behaved in a significantly more ductile manner post the ultimate limit state than the steel bar reinforcement. Their differences are quantified in terms of toughness, defined as the area under the bond-slippage curve [20]. The toughness was calculated until 30 mm of slippage for each specimen. Figure 8 shows the toughness values of the present specimens.

However, by comparing the peak pullout loads of the five specimens against the corresponding yield loads of the steel reinforcements shown in Table 1, it can be seen that the plate reinforced specimens failed at loads well below the latter, in contrast to the deformed bar reinforced specimen.

It would therefore appear from the present beam-end tests that the chequer steel plates did not have adequate bond strength to enable themselves to yield when used as horizontal reinforcements in concrete beams. However, this apparent indication is inconsistent with the test results of Hadi et al. [35] for steel plate reinforced concrete beams subjected to four-point bending tests. The four-point bending tests demonstrated that, not only the chequer steel plate reinforced beams attained similar yield moments to the deformed bar reinforced beam, but also exhibited much more ductile post-ultimate behaviour. In the four-point bending tests [35], the deformed steel bars had a similar yield load to that of the chequer steel plates.

5. Conclusions

This technical note has described an experimental study to investigate the bond behaviour of steel plate reinforcements in concrete members. The following findings can be summarised:

1. The general failure mode of beam-end specimens was pullout accompanied by splitting crack. Only the specimen reinforced with a smooth steel plate had a simple pullout failure without visible cracks.
2. The lozenges of chequer steel plate increased the pullout load by 80% compared with the smooth steel plate.
3. The existence of steel bolt (welded to the chequer steel plate) increased the pullout load by 28%.
4. The steel plate reinforced specimens had much less slippage prior to the ultimate limit state compared to the deformed steel bar reinforced specimen. The steel plate reinforced specimens had much better toughness than the deformed steel bar reinforced specimen. The reason is that the bond area of each steel plate was much larger than that of the steel bar.
5. The thickness of the concrete cover can have a significant effect on the pullout failure load of the steel plate reinforced specimen.
6. The existing equations cannot be used to estimate the bond strength of the steel plate reinforcements.
7. The pullout failure loads of the beam-end specimens with steel plate reinforcements were much lower than the corresponding yield loads of the reinforcements, in contrast to the case of the deformed steel bar specimen.
8. The beam-end method may not be an appropriate method for comparing the bond performance between a chequer steel plate and a steel bar, used as tensile reinforcements in a concrete beam subjected to bending.

Acknowledgements

The authors wish to thank Messrs Cameron Neilson, Ritchie McClean, Fernando Escribano, and Alan Grant for their help in conducting the experimental work. The first author would also like to thank the Iraqi government for the financial support of his Ph.D scholarship.

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Table 1: Test matrix

Test specimen	Embedment	Yield load of reinforcement (kN)	Steel bolts	
			Diameter (mm)	Length (mm)
BE-N20	N20	170	---	
BE-HP	Horizontal chequer steel plate	370	---	
BE-HSP[a]			---	
BE-HBP[b]			20	100
BE-VP	Vertical chequer steel plate	370	---	

[a] The lozenges were removed.

[b] A steel bolt was welded to the chequer steel plate.

Table 2: The pullout forces and bond strengths of specimens

Test specimen	Pull-out force (kN)	Measured bond strength (MPa)	Calculated bond strength by Zuo and Darwin (MPa)	Calculated bond strength by ACI-408R-03 (MPa)
BE-N20	176	12.4	11.4	11.2
BE-HP	99	2	5.2	5.2
BE-HSP	55	1.1	--	--
BE-HBP	127	2.6	--	--
BE-VP	199	4	5.2	5.2

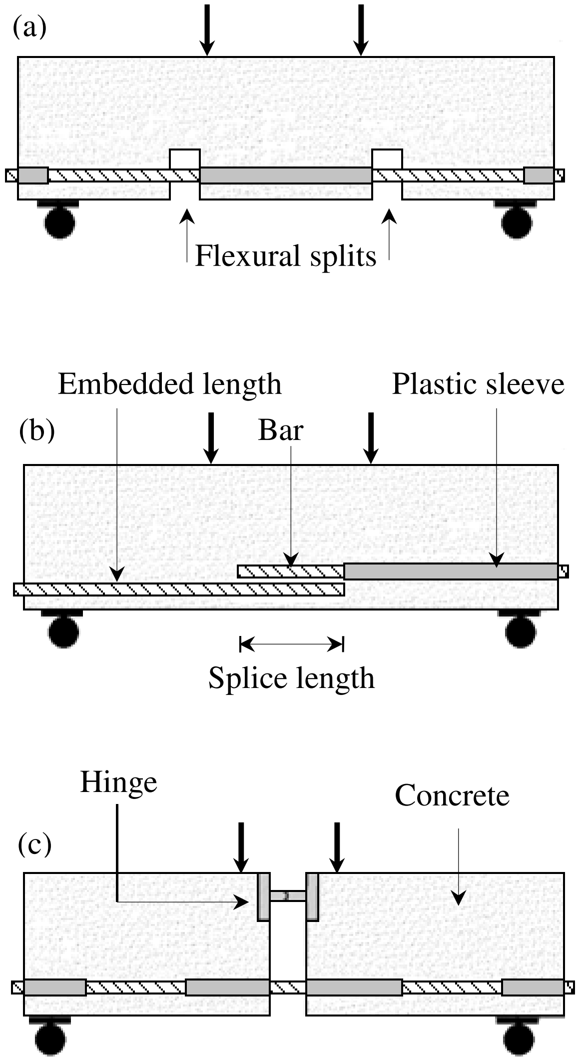


Figure 1: Bond test methods: (a) Anchorage beam [4]; (b) Splice beam [4]; and (c) Beam-bending [22]

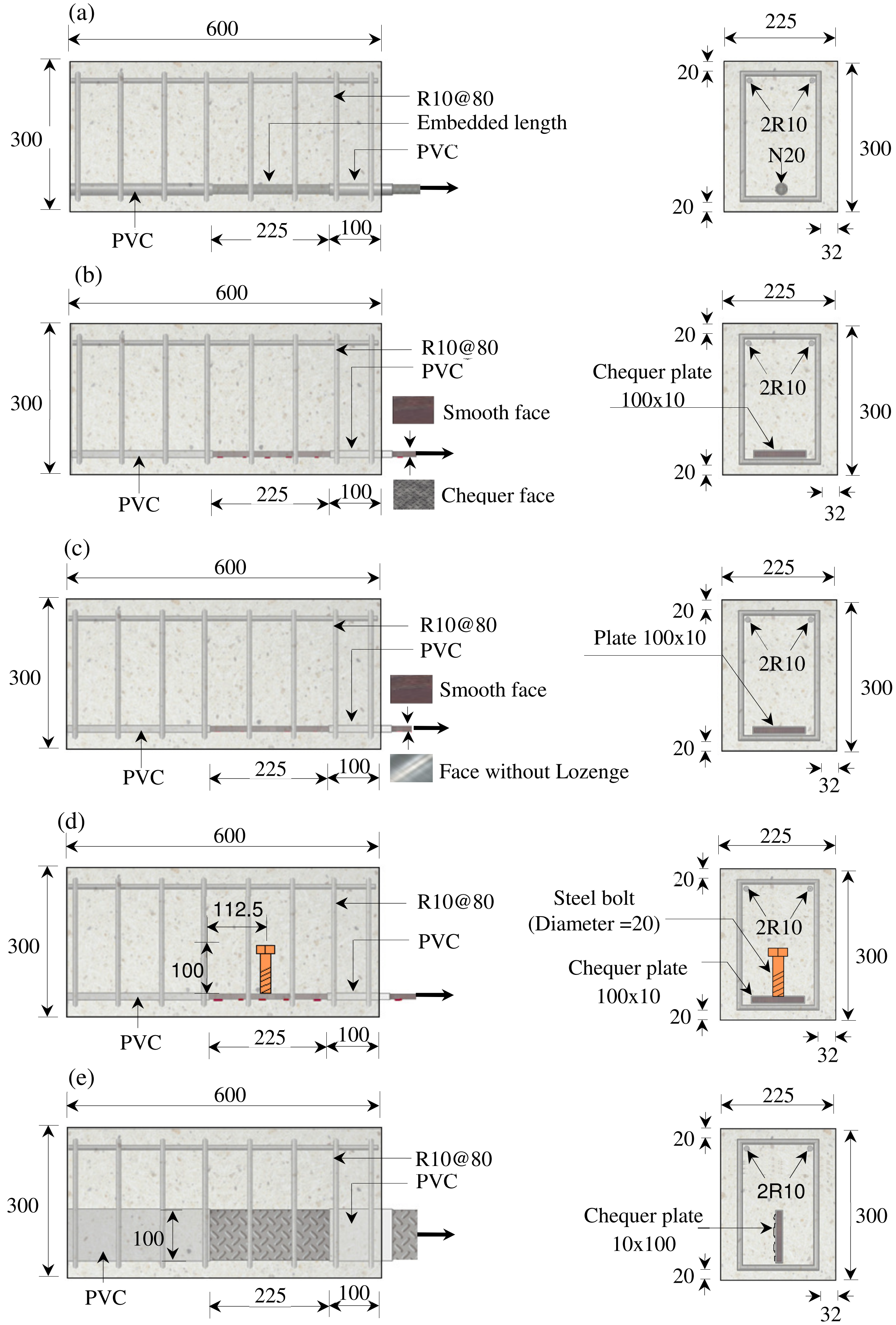


Figure 2: Beam-end specimens: (a) BE-N20; (b) BE-HP; (c) BE-HSP; (d) BE-HBP; (e) BE-VP

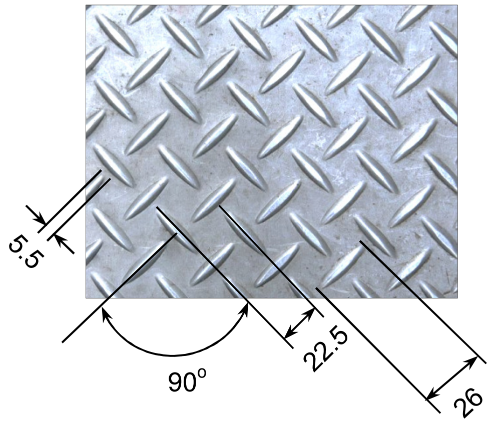


Figure 3: Geometry of lozenges in chequer steel plates

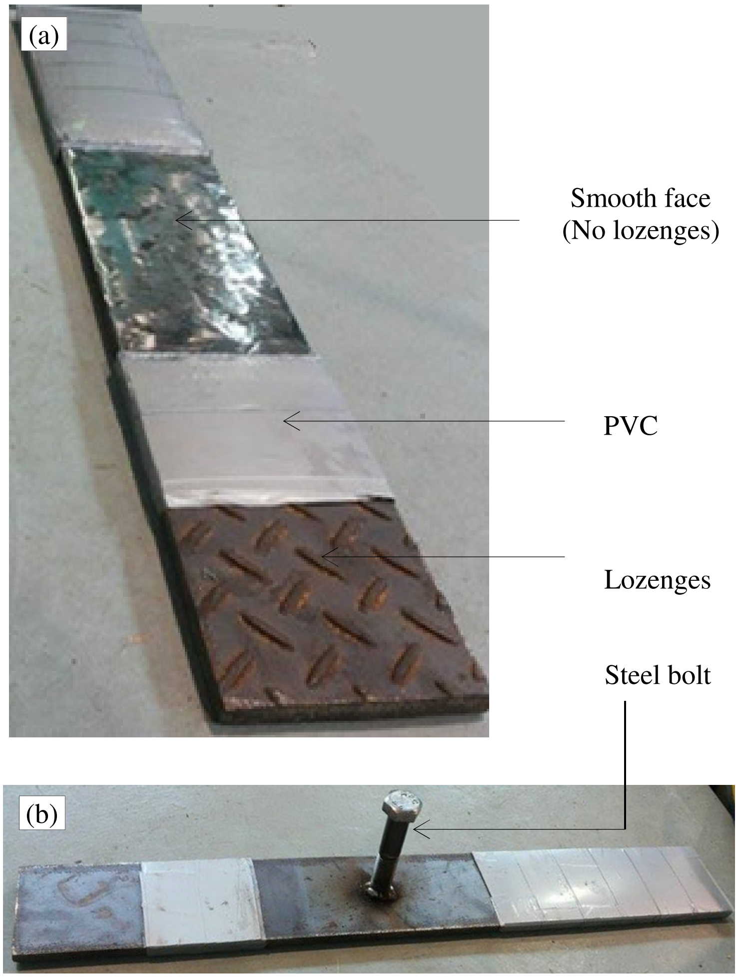


Figure 4: (a) Chequer steel plate with removed lozenges for Specimen BE-HSP; (b) Steel bolt welded to chequer steel plate for Specimen BE-HBP

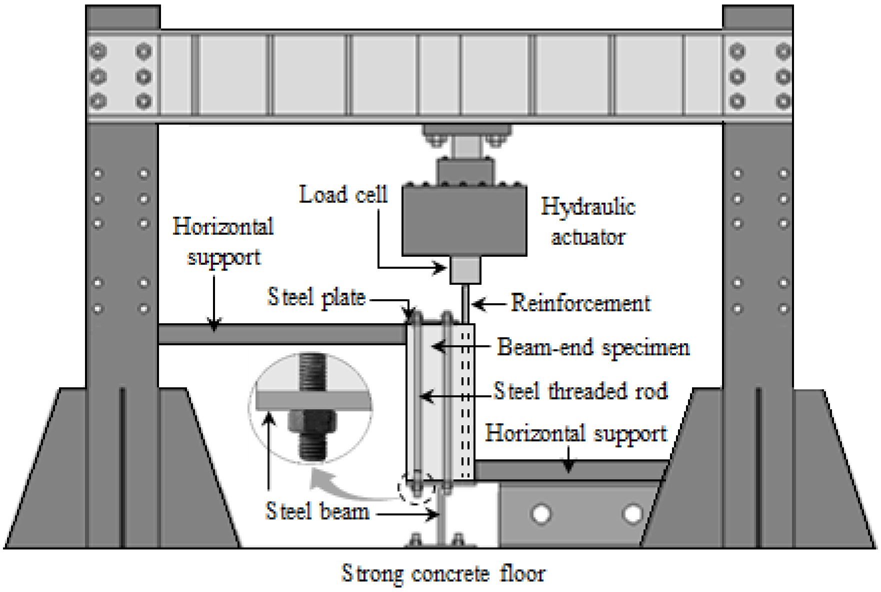


Figure 5: Test setup

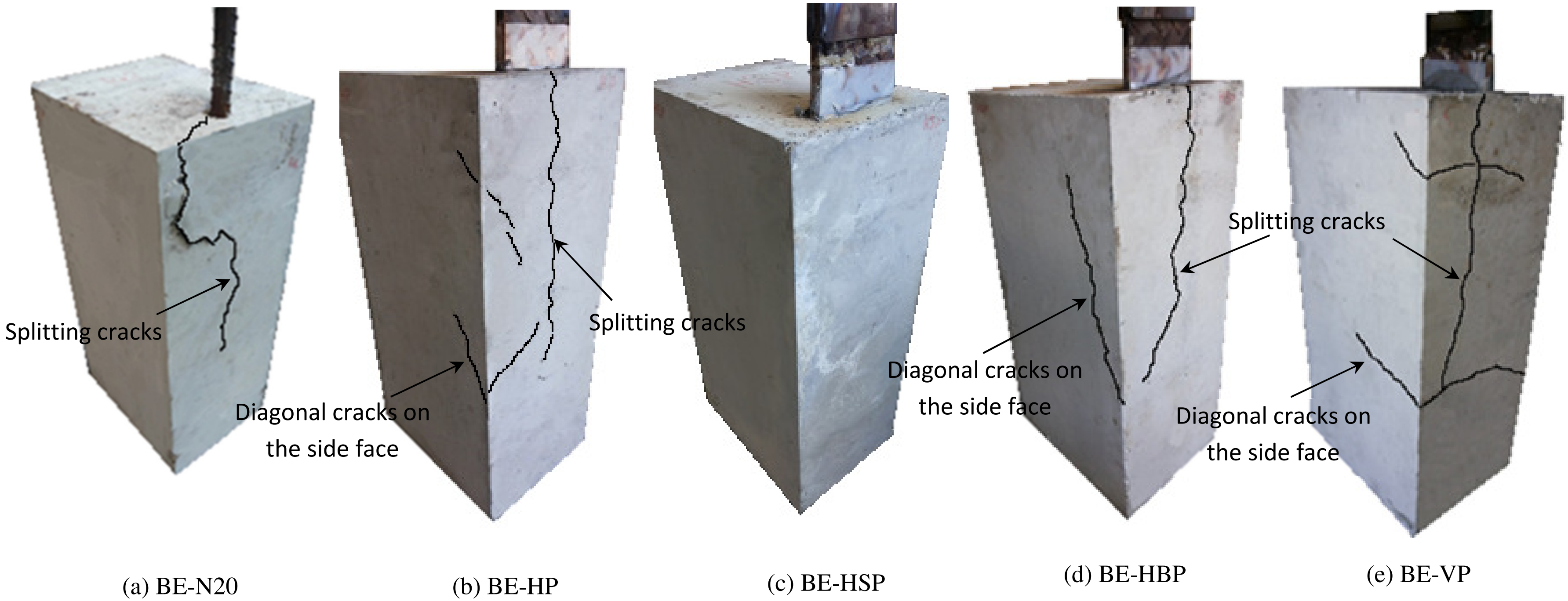


Figure 6: Failure modes of beam-end pullout specimens: (a) BE-N20; (b) BE-HP; (c) BE-HSP; (d) BE-HBP; and (e) BE-VP

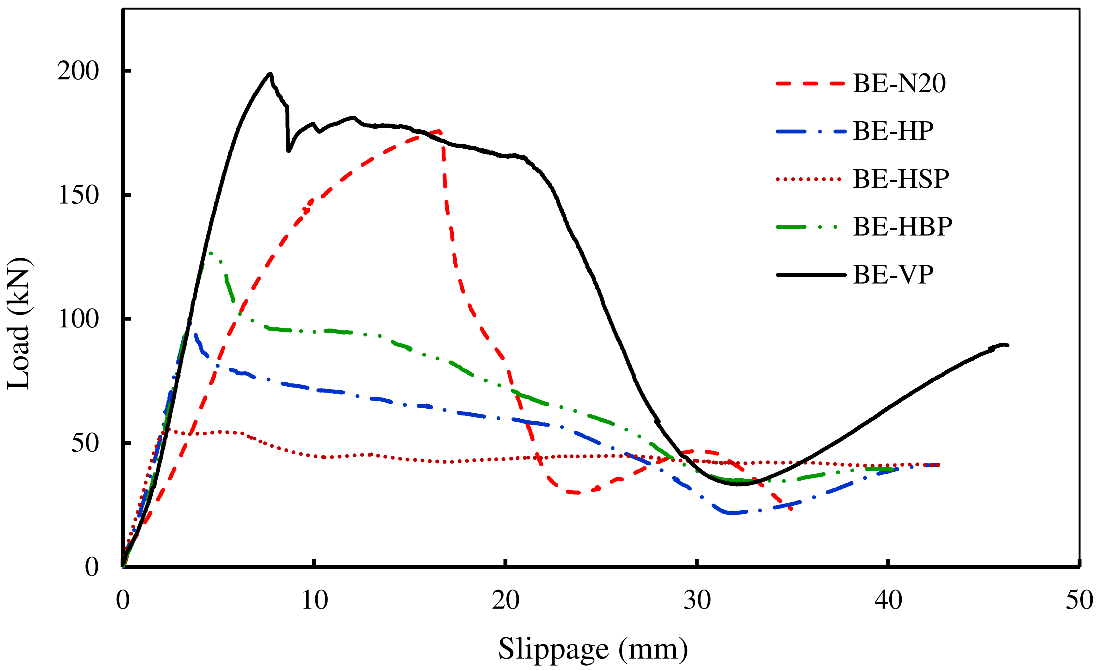


Figure 7: Load-slippage curves of beam-end pullout specimens

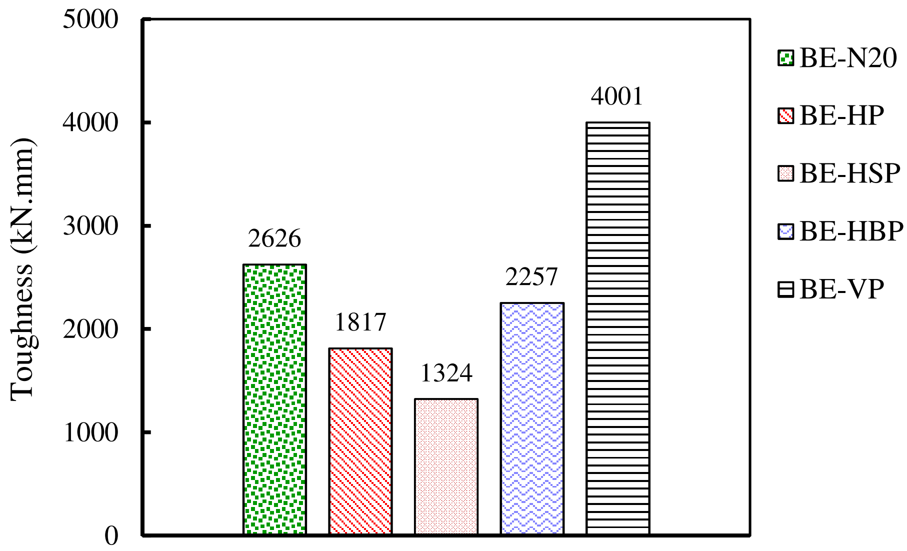
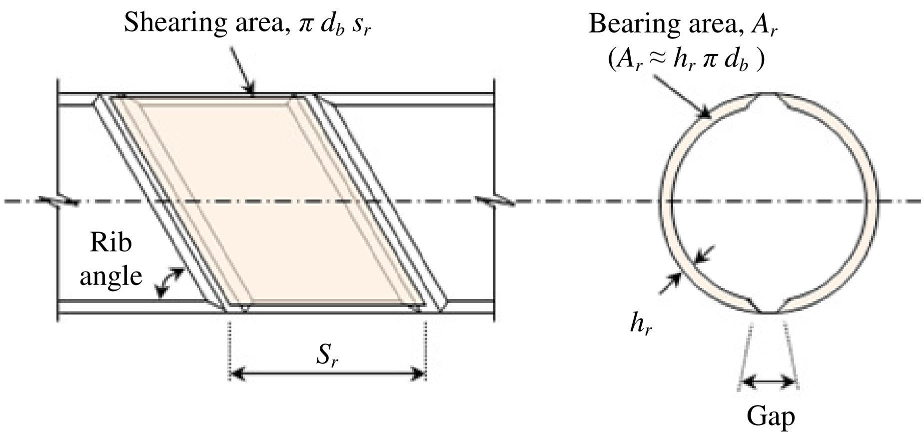


Figure 8: Toughness of beam-end pullout specimens



R_r = (bearing area/Shearing area) ≈ h_r/S_r)

Figure 9: Definition of relative rib area of the steel bar reinforcement (*R_r*)